Design of Single Plate Shear Connections

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INTRODUCTION

Single plate shear connections, often referred to as shear tabs, have gained considerable popularity in recent years due to their efficiency and ease of fabrication. Shear tab connections are primarily used to transfer beam end reactions to the supporting elements. The connection consists of a plate welded to a support at one edge and bolted to a beam web. Figure 1 shows typical applications of single plate shear connections. This paper presents the summary of a research project on the behavior and design of single plate shear connections. Based on experimental and analytical studies, a new design procedure is developed and presented.

The AISC–ASD[15] as well as AISC–LRFD[16] specifications have the following provisions with regard to shear connections:

> "Except as otherwise indicated by the designer, connections of beams, girders, or trusses shall be designed as flexible, and may ordinarily be proportioned for the reaction shears only.
>
> "Flexible beam connections shall accommodate end rotations of unrestrained (simple) beams. To accomplish this, inelastic action in the connection is permitted."

Steel shear connections not only should have sufficient strength to transfer the end shear reaction of the beam but according to above provisions, the connections should also have enough rotation capacity (ductility) to accommodate the end rotation demand of a simply supported beam. In addition, the connection should be sufficiently flexible so that beam end moments become negligible. Thus, like any shear connection, single plate shear connections should be designed to satisfy the dual criteria of shear strength and rotational flexibility and ductility.

Shear-Rotation Relationship in a Shear Connection

To investigate the behavior and strength of a shear connection, it is necessary that realistic *shear forces and their corresponding rotations* be applied to the connection. In an earlier research project,[2] the shear-rotation relationship for the end supports of simply supported beams was studied. A computer program was developed[1] and used to simulate increased monotonic uniform loading of the beams supported by simple connections until the beams collapsed.[1,2]

The studies indicated that the relationship between the end shear and end rotation is relatively stable and depends primarily on the shape factor Z_x/S_x of the cross section, L/d of the beam and the grade of steel used. Figure 2 shows a series of curves representing shear forces and corresponding rotations that will exist at the ends of simply supported beams. The curves correspond to beams of A36 steel having cross sections from W16 to W33 and L/d ratios of 4 to 38. Also shown in Fig. 2 is a tri-linear curve "abcd" suggested to be a realistic representative of the shear-rotation curves. The tri-linear curve "abcd" is proposed to be used as a standard load path in studies of shear connections. Curve "abcd" is used instead of the more conservative curve "aef" because it is felt that curve "abcd" represents a more realistic maximum span-to-depth ratio for most steel structures. For special cases of very large span-to-depth ratio or high strength steels, the rotational demand may be greater than that of curve "abcd". For such cases special care must be taken to assure the rotational ductility demand of the beam is supplied by the connection.

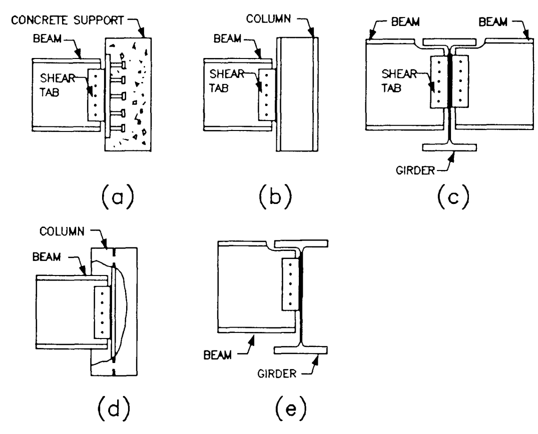

Fig. 1. Typical Single Plate Shear Connections

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The shear-rotation curves plotted in Fig. 2 are established based on the assumption of elastic-perfectly-plastic bending moment capacity for the beam. To include the effect of strain hardening, the segment "cd" in curve "abcd" is included.

The behavior of shear connections has been studied in the past by several investigators.[8,10-12] However, in most cases, the shear connections have been subjected to moment and rotation or only direct shear without rotation instead of a realistic combination of shear and rotation. Figure 3 shows the shear rotation relationships that existed in several studies including this research project.

EXPERIMENTAL RESEARCH

In order to identify limit states of strength and to verify the validity of the design procedures that were developed and proposed, five full scale beam-to-column connection assemblies were tested. A summary of the experimental studies follows. More detailed information on the research project can be found in References 3 and 6.

Test Set-up

The test set-up shown in Fig. 4 was used to apply shear-rotation relationship of curve "abcd" in Fig. 2 to the specimens.

The main components of the test set-up were a computer based data acquisition and processing system, two actuators R and S and support blocks. Actuator S, which was close to the connection, was force controlled and provided the bulk of the shear force in the connection. Actuator R, which was displacement controlled, provided and controlled the beam end rotation.

Test Load Path

The proposed standard shear-rotation relationship shown as curve "abcd" in Fig. 2 was applied to the connections in all of the test specimens. To establish the curve, coupon tests of the plate material were conducted prior to connection tests and the yield point and ultimate strength of the plate material were obtained. The shear yield capacity of the single plate in each test specimen was calculated by multiplying the von Miess criterion of shear yield stress, $1/\sqrt{3}F_y$, by the shear area of the plate. The shear yield capacity of the plate, denoted as R_y, was taken as equal to the shear at point "c" of curve "abcd" in Fig. 2. Thus the shear yield capacity of the shear tab was assumed to occur when the moment at midspan was equal to M_p. As a result, a corresponding M_p can be calculated for each connection to be equal to $R_yL/4$. The end rotation of the beam when midspan moment reached M_p was set equal to 0.03 radians.

To establish point "b" in curve "abcd", the shear at this point was set equal to $4M_y/L$ and the rotation was set equal to 0.02 radian. This implies that when beam midspan moment reaches M_y, the end rotation will be equal to 0.02 radian. The value of M_y, the end rotation will be equal to 0.02 radian. The value of M_y for each specimen was calculated by dividing M_p by the shape factor. A shape factor of 1.12 was used in all specimens.

Segment "cd" in Fig. 2 corresponds to strain hardening of the beam and the increased moment at beam midspan which results in increased shear at the beam ends. To establish "cd", it was assumed that when the midspan moment reaches a value of $(F_u/F_y)M_p$, the beam end rotation will be equal to 0.1 radian.

In summary, load path "abcd" in Fig. 2 reflects the behavior of the beam and its effect on connection shear and rotation. Segment "ab" corresponds to the elastic behavior of beam. At point "b", midspan moment of the beam reaches M_y and the beam softens. Segment "bc" corresponds to inelastic behavior of the beam. At point "c", the midspan moment reaches M_p. Segment "cd" represents extra beam capacity that can develop due to beam strain hardening.

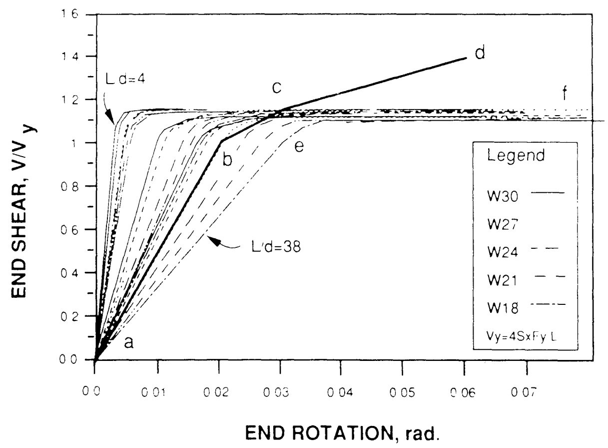

Fig. 2. Shear-Rotation Relationship for Ends of Simple Beams

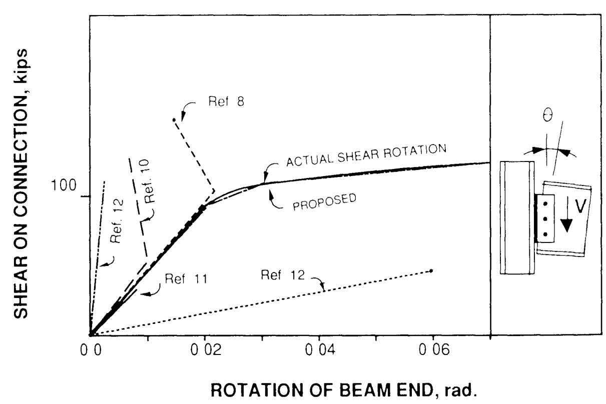

Fig. 3. Shear-Rotation Relationship used in Several Studies

Table 1. Properties of Test Specimens									
TEST GROUP	TEST NO.	NO. OF BOLTS	DIA. OF BOLTS	TYPE OF BOLTS*	PLATE DIMENSIONS	EDGE DISTANCE	‡ ACTUAL WELD SIZE	BEAM MATERIAL	PLATE MATERIAL
			in.		in. × in. × in.	in.	in.		
ONE	1	7	3/4	A325-N	21 × 3/8 × 4-1/4	1-1/2	1/4	A36	A36
	2	5	3/4	A325-N	15 × 3/8 × 4-1/4	1-1/2	1/4	A36	A36
	3	3	3/4	A325-N	9 × 3/8 × 4-1/4	1-1/2	1/4	A36	A36
TWO	4	5	3/4	A490-N	14-1/4 × 3/8 × 3-7/8	1-1/8	7/32	Gr. 50	A36
	5	3	3/4	A490-N	8-1/4 × 3/8 × 3-7/8	1-1/8	7/32	Gr. 50	A36

**All bolts were tightened to 70% of proof load. In all specimens diameter of bolt hole was 1/16 inch larger than nominal diameter of bolt. "N" indicates that in all specimens threads were included in shear plane.*
‡Size of all welds was specified as 1/4 inch.

Test Specimens

Each test specimen consisted of a wide flange beam bolted to a single plate shear connection which was welded to a column flange as shown in Fig. 1b. The properties of the test specimens were selected in consultation with a professional advisory panel. These properties are given in Table 1. The bolt holes in all specimens were standard round punched holes. All bolts were tightened to 70% of proof load using turn-of-the-nut method.[13,14] All shear tabs were cut from a single piece of steel. The yield stress and ultimate strength for material of shear tabs were 35.5 ksi and 61 ksi respectively. The condition of faying surfaces was clean mill scale. The electrodes were equivalent of E7018.

The bolt spacing in all specimens was 3 in. The edge distance in the horizontal as well as vertical direction for specimens 1,2 and 3 was 1-1/2 in. (two times diameter of bolt) and for specimens 4 and 5 was 1-1/8 in. (1.5 times diameter of bolts).

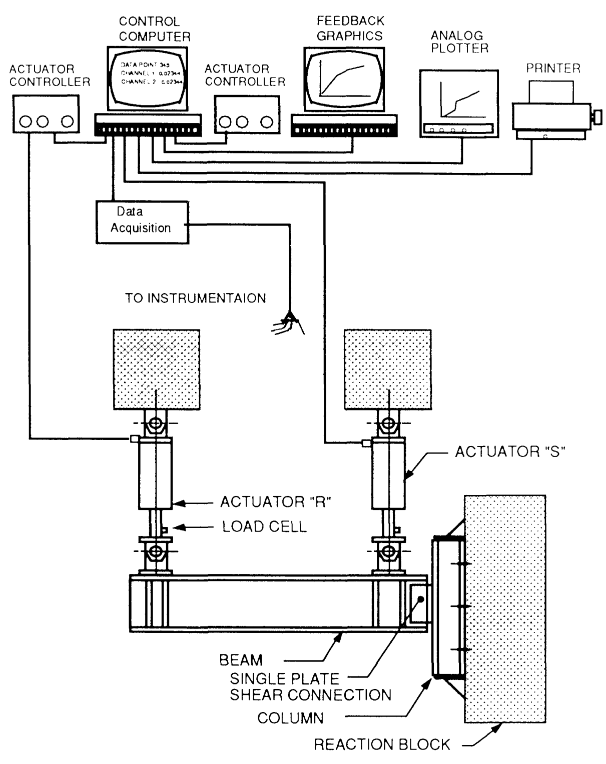

Fig. 4. Test Set-up Used in Experiments

Behavior of Test Specimens

The experiments were conducted in two groups as indicated in Table 1. The main differences of specimens in these two groups were the type of bolt (A325 or A490), material of beam (A36 or grade 50) and edge distance ($2d_b$ or $1.5d_b$). The behavior of specimens in the two groups is summarized in the following sections.

Behavior of Specimens 1,2 and 3 (Group One)

Specimens 1,2 and 3 showed very similar behavior throughout the loading. The most important observation was the significant inelastic shear deformations that took place in all three specimens as shown in Fig. 5.

All test specimens failed due to sudden shear fracture of the bolts connecting the single plate to the beam web as shown in Fig. 6a. The examination of bolts after failure indicated that the A325 bolts in these specimens had developed significant permanent deformations prior to fracture as indicated in Fig. 6b. In these three specimens the welds did not show any sign of yielding other than in specimen 3 which showed minor yielding at the top and bottom of welds prior to fracture of bolts.

A study of the bolt holes after the completion of tests 1,2 and 3 indicated that permanent bearing deformations had taken place in the plate as well as in the beam web. The magnitude of the deformations in the plate and beam bolt holes were almost equal but in opposite directions. The deformations of the plate bolt holes, drawn to scale are shown in Fig. 7. The arrows indicate the direction of the movement of the bolts which is expected to be approximately the direction of the applied force due to shear and moment. It is interesting to note that nearly vertical orientations of arrows indicate the presence of a large vertical shear accompanied by a relatively small moment in the connections.

Behavior of Specimens 4 and 5 (Group Two)

The behavior of specimens 4 and 5 was similar to the previous three tests. However, shear yielding of the plate was more apparent. Specimen 4 failed due to shear fracture of bolts in a manner similar to previous tests shown in Fig. 6a. In addition, minor yielding was observed on the weld lines of this specimen. Specimen 5 failed by almost simultaneous fracture of weld lines and bolts as shown in Fig. 8. It appears that at the time of failure, weld lines started to fracture first while bolts were on the verge of fracture. When sudden fracture of welds occured the resulting shock caused fracture of the bolts which appeared to be almost simultaneous with weld fracture. Bolts in specimens 4 and 5 were A490 bolts. An examination of the bolts after fracture showed less permanent deformations in these bolts than the A325 bolts used in previous three tests (see Fig. 6b).

Study of bolt holes in the shear tabs of specimens 4 and 5 indicated that significantly larger bolt hole deformations had occured in these two specimens compared to specimens 1,2 and 3. However, the bolt holes in the beam web in specimens 4 and 5 had only minor permanent deformations.

In summary, based on observations made during the tests, it appears that shear tabs go through three distinctive phases of behavior. At the very early stages, a shear tab acts as a short cantilever beam with moment being dominant. As load increases, the shear tab acts as a deep shear beam with the shear yielding effect dominant (as in specimens 1 through 4). If bolts and welds do not fail during the shear phase, because of large deformations, the shear tab acts similarly to the diagonal member of a truss and carries the applied shear by a combination of shear and diagonal tension effects (as in specimen 5).

Experimental Data

The results of experiments at the time of failure are summarized in Table 2.

DISCUSSION OF EXPERIMENTAL RESULTS

Shear Yielding of Single Plate

The yielding of the single plate was primarily due to shear stresses and was quite ductile. It was evident that considerable shear yielding occurred in the plate between the bolt line and weld line. The shear yielding was almost uniformly distributed throughout the depth of the plate as measured by strain gages that were attached to the plates.[3,6] Therefore, in the proposed design procedure discussed later, the shear capacity of plate is calculated by multiplying gross area of plate by uniformly distributed shear stresses.

In specimen 3, at later stages of loading and after significant shear yielding, the bottom portion of the shear tab showed signs of minor local buckling as shown in Fig. 6a. This local buckling was attributed primarily to loss of stiffness of plate material due to shear yielding. Until this phe-

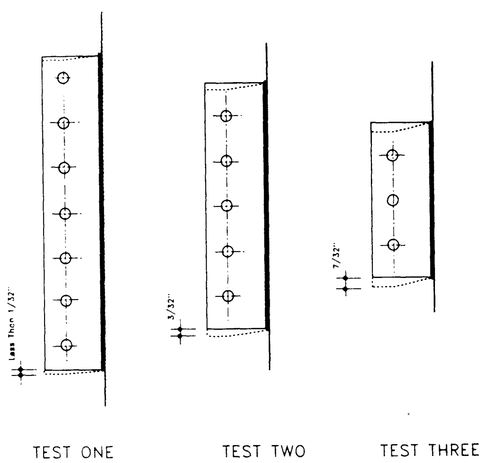

Fig. 5. Plate Shear Deformations in Specimens 1,2,3

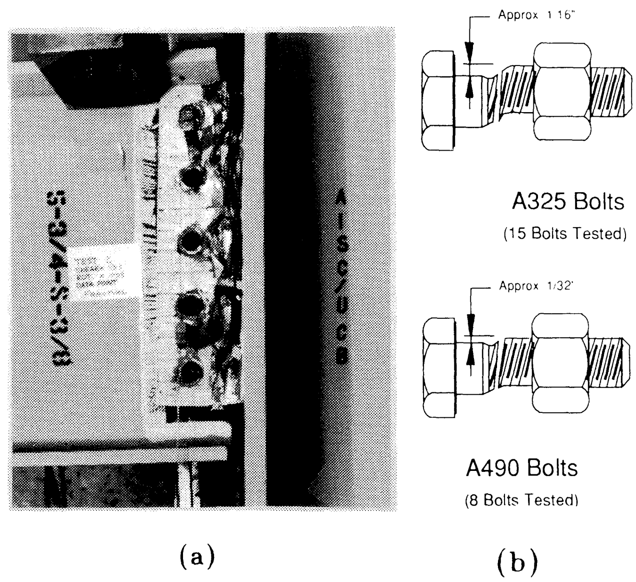

Fig. 6. Typical Bolt Failure of Test Specimens

Table 2. Results of shear strength Tests									
Specimen			Observed Failure Mode	Connection Response					
Test Group	Test No.	No. of Bolts		Shear Displacement	Shear Force	Beam End Rotation	‡ Moment at Bolt Line	‡ Moment at Weld Line	Maximum Moment at Weld Line
				in.	kips	rad.	kip in.	kip in.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
One	1	7	Bolts Fractured	0.27	160	0.026	306	745	1028
	2	5	Bolts Fractured	0.34	137	0.054	314	691	734
	3	3	Bolts Fractured	0.46	94	0.056	20	279	350
Two	4	5	Bolts Fractured	0.35	130	0.053	273	631	686
	5	3	Welds and Bolts Fractured	0.52	79	0.061	−47	170	237

**In some cases like these, moment decreased as shear and rotation increased.*
‡Positive moments cause top of connection to be in tension.

nomenon is studied thoroughly, it is suggested that local buckling be avoided. To prevent local buckling, it is recommended that the distance between the bolt line and the weld line be less than ½ of the plate length.

Fracture of Net Area of Plate

In the single plate specimens that were tested, the net area of the plate did not fracture. Only specimen 5 showed signs of approaching fracture of net section. Nevertheless, this failure mode has been observed in similar cases in several experiments on tee framing connections.[4,5] The stem in a tee framing connection behaves similarly to a shear tab. The formula currently used in calculating net area in shear fracture is:[15]

$$A_{ns} = A_{vg} - n(d_b + 1/16)t_p \qquad (1)$$

The studies of tee connections indicated that the shear fracture occurred consistently by fracture of net section along the edge of the bolt hole and not along the centerline of bolts. It was suggested that[4,5] the net area effective in shear be equal to the average of net area along the bolt centerline and the gross area. Using the suggested method to calculate net area in shear, the effective net area in shear can be written as:

$$A_{nse} = A_{vg} - (n/2)(d_b + 1/16)t_p \qquad (2)$$

Shear-Rotation Behavior

Figure 9 shows the actual shear-rotation relationship that was recorded during each test. It is observed that the rotational ductility of the connections increased as the

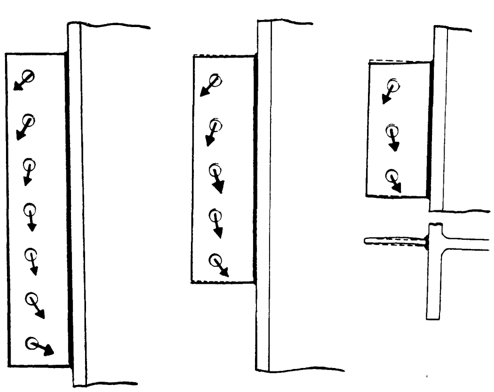

Fig. 7. Plate Bolt Hole Deformations after Tests

Fig. 8. Failure of Welds and Bolts in Specimen 5

number of bolts decreased. The rotational ductility of the connection in specimen 1 with 7 bolts was 0.026 radians which was about half the rotational ductility of the connections in specimens 2, 3, 4 and 5 with three or five bolts, all of which were able to reach rotations in excess of 0.05 radians.

Movement of Point of Inflection

Figure 10 shows movement of point of inflection of the beam toward the support as the shear force was increased. Even under relatively small load, in all specimens, the point of inflection moved toward the support and remained almost stationary for the remainder of each test.

Using experimental data, the following empirical equation was developed to define the location of the point of inflection for test specimens.

$$e = (n-1)(1.0), \text{ in.} \tag{3}$$

where n is the number of bolts used in the connection, and e is the distance of point of inflection from the support (i.e. from the weld line).

It is important to realize that in the experiments reported here, the columns were fixed to supports and rigid body rotation of the connections was prevented. If due to frame action or other causes, the support to which a shear tab is connected rotates, due to rigid body rotation, the location of point of inflection may be affected. However, the concurrent values of shear and moment acting on the shear tab at any given time cannot exceed the values obtained from plasticity conditions (interaction curves) of plate for shear and moment.

Behavior and Design of Bolts

In all specimens, an examination of bolts and bolt holes after failure indicated that bolt shanks had experienced considerable shear deformations before failure.

Studies on the behavior of single bolts in shear[11] have indicated that for A325 bolts and A36 plate, if the thickness of the plate is not greater than ½ times the diameter of the bolt, considerable but tolerable bolt hole deformations will take place. The limited bolt hole deformations are desirable since they increase rotational flexibility and ductility of the connections. In studies of tee connections[4,5] in three specimens, ½ in. thick tee stems were used with ⅞ in. diameter bolts. The behavior of these tee specimens indicated that even when thickness of stem was equal to d_b + 1⁄16 in., desirable bearing deformations took place in the bolt holes. Therefore, based on these studies, and to obtain flexible and ductile single plate connections, the thickness of the plate is recommended to be less than or equal to ½ of the bolt diameter plus 1⁄16 in.

An examination of the deformations of bolts and bolt holes at the completion of the tests indicated that the bolts were primarily subjected to direct shear accompanied by a small moment (see arrows in Fig. 6a).

As Fig. 10 indicates, the point of inflection for test specimens was almost stationary, fluctuating between an eccentricity of n and $n - 1$ in. At the time of failure of the bolts in all specimens, the location of the point of inflection was close to $n - 1$ in. Therefore, it is recommended that bolts be designed for combined effects of direct shear and a moment equal to the shear multiplied by the eccentricity of the bolt line from point of inflection given by:

$$e_b = (n-1)(1.0)-a \tag{4}$$

where,

a = distance between the bolt line and weld line,

e_b = distance from the point of inflection to the bolt line.

Behavior and Design of Welds

Table 2 gives values of shear and moment at failure for each test. The fillet welds mainly experienced a direct

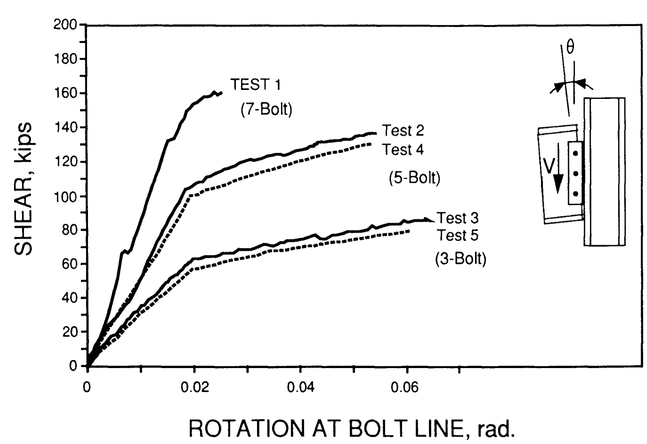

Fig. 9. Shear-Rotation Curves for Test Specimens

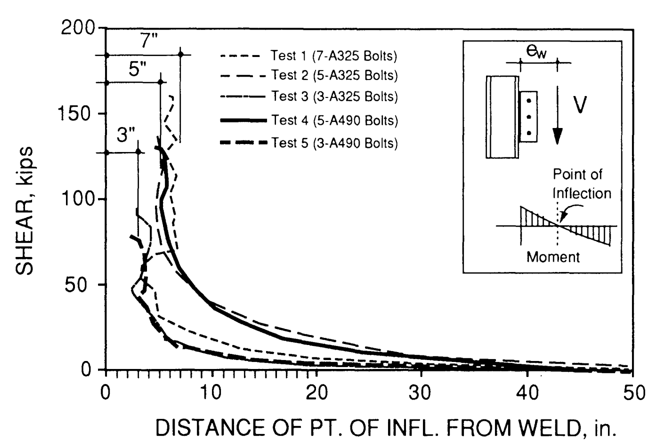

Fig. 10. Movement of Point of Inflection

shear accompanied by a relatively small moment. The strain measurements adjacent to the welds also supported this conclusion.[3,6] Therefore, fillet welds are recommended to be designed for the combined effects of shear and a small bending moment.

The main goal of the proposed design procedure is to ensure yielding of shear tab prior to failure of welds. In order to achieve this goal the welds should be designed to be stronger than the plate. Thus, the design shear force acting on the welds is recommended to be equal to the shear capacity of the plate and not the applied shear force. Therefore, the maximum shear force acting on the weld is equal to $1/\sqrt{3}\ F_y L_p t_p$. In Allowable Stress Design, the design shear force for welds is equal to $0.40 F_y L_p t_p$. The moment acting on the weld is equal to shear force multiplied by the eccentricity of the point of inflection from the weld line. To be conservative, it is recommended that the eccentricity of the point of inflection from the weld line be equal to n inches,

$$e_w = (n)(1.0) \tag{5}$$

Since the design of welds in the proposed method is a capacity design, it is not necessary to use welds that can resist forces much greater than the plate capacity. As part of phase two of this investigation, a study was conducted to establish minimum and maximum weld requirements to develop the strength of single plate. The study indicated that for A36 plate and E70 electrodes the weld size need not be more than $0.75t_p$ and should not be greater than t_p. The upper limit of t_p on the weld size was imposed to prevent excessive welding of the plate which will be costly and might cause heat damage to the plate without achieving extra strength in the connection.

Moment-Rotation Curves

Moment-rotation curves for the test specimens are shown in Fig. 11. Moments and rotations were measured along the bolt line. As the plots indicate, connections with fewer bolts developed smaller moments and exhibited larger rotational ductility. During the elastic range of behavior, moment increased with shear. As the load increased, due to connection deformations, rotational stiffness and bending moment decreased and then gradually increased at a much smaller rate. The decrease is attributed to slips and inelastic deformations in the connections and the increase is attributed to strain hardening.

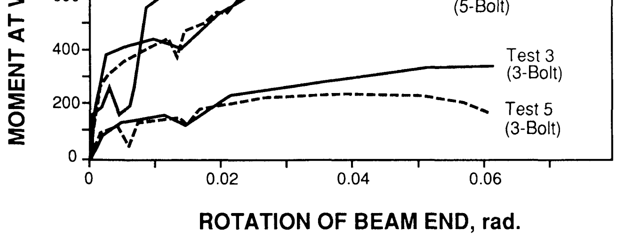

Fig. 11. Moment-Rotation Curves for Test Specimens

PROPOSED DESIGN PROCEDURE

The following design procedure is based on the analyses of the experimental results and the information available on the actual behavior of shear connections.[1-6,9]

General Requirements

The single plate framing connections covered by these procedures consist of a plate bolted to a beam web and welded to a support on one edge of plate.

In design of a single plate framing connection, the following requirements should be satisfied:

1. The connection has only one vertical row of bolts and the number of bolts is not less than 2 or more than 7.
2. Bolt spacing is equal to 3 in.
3. Edge distances are equal to or greater than $1.5d_b$. The vertical edge distance for the lowest bolt is preferred not to be less than 1.5 in.
4. The distance from bolt line to weld line is equal to 3 in.
5. Material of the shear plate is A36 steel to facilitate yielding.
6. Welds are fillet welds with E70xx or E60xx electrodes.
7. Thickness of the single plate should be less than or equal to $d_b/2 + 1/16$.
8. The ratio of L_p/a of the plate should be greater than or equal to 2 to prevent local buckling of plate.
9. ASTM A325 and A490 bolts may be used. Fully tightened as well as snug tight bolts are permitted. The procedure is not applicable to oversized or long slotted bolt holes. Standard or short-slotted punched or drilled holes are permitted.

Consideration of Limit States in Design

The following limit states are associated with the single plate framing connections.

1. Shear failure of bolts.
2. Yielding of gross area of plate.
3. Fracture of net area of plate.
4. Fracture of welds
5. Bearing failure of beam web or plate.

Shear Failure of Bolts

Bolts are designed for the combined effects of direct shear and a moment due to the eccentricity e_b of the reaction from the bolt line. The eccentricity e_b for single plate connections covered by these procedures can be assumed to be equal to 3 in., which is the distance from bolt line to weld line. The value is conservative when the single plate is welded to a rigid support. The value is more realistic when the supporting member is a relatively flexible element.

More realistic values for e_b can be calculated from the following equations:

if single plate is welded to a rotationally rigid element, e_b is obtained from:

$$e_b = (n-1)(1.0)-a \tag{6}$$

if single plate is welded to a rotationally flexible element, e_b is larger value obtained from:

$$e_b = \text{Max} \left| \begin{array}{ll} (n-1)(1.0) - a & (7a) \\ a & (7b) \end{array} \right.$$

where,
n = number of bolts
a = distance from bolt line to weld line, in.
e_b = eccentricity, in.

By using methods outlined in Reference 7 including using Tables X of the AISC-ASD Manual[13] the bolts are designed for the combined effects of shear R, and moment equal to Re_b.

Yielding of Gross Area of Plate

The equation defining this limit state in allowable stress design (ASD) format is:

$$f_{vy} \leq F_{vy} \tag{8}$$

where,

$$f_{vy} = R \,/\, A_{vg} \tag{9}$$

$$F_{vy} = 0.40\ F_y \tag{10}$$

$$A_{vg} = L_p\ t_p \tag{11}$$

Fracture of Net Area of Plate

The equation defining this limit state in allowable stress design (ASD) format is:

$$f_{vu} \leq F_{vu} \tag{12}$$

where,

$$f_{vu} = R \,/\, A_{ns} \tag{13}$$

$$F_{vu} = 0.30\ F_u \tag{14}$$

$$A_{ns} = [L_p - n(d_b + 1/16)]t_p \tag{15}$$

If the beam is coped, the block shear failure of the beam web also should be considered as discussed in the AISC-ASD Specification.[15]

Weld Failure

The welds connecting the plate to the support are designed for the combined effects of direct shear and a moment due to the eccentricity of the reaction from the weld line, e_w. The eccentricity e_w is equal to the larger value obtained from:

$$e_w = \text{Max} \left| \begin{array}{ll} (n)(1.0) & (16a) \\ a & (16b) \end{array} \right.$$

where,
n = number of bolts
e_w = eccentricity, in.
a = distance from bolt line to weld line, in.

By using methods outlined in Reference 7 including using Tables XIX of the AISC-ASD Manual,[13] the fillet welds are designed for the combined effects of shear equal to R and moment equal to Re_w.

Bearing Failure of Plate or Beam Web

To avoid reaching this limit state, it is recommended that the established rule of horizontal and vertical edge distances equaling at least 1.5 the bolt diameter be followed. The bolt spacings should satisfy requirements of the AISC-ASD Specification.[15] The bearing strength of connection can be calculated using the provisions of the AISC-ASD Specification.[15]

Summary of Design Procedure

The following steps are recommended to be taken in design of single plate framing connections:

1. Calculate number of bolts required to resist combined effects of shear R, and moment Re_b using Table X of the AISC-ASD Manual.[13]
If the single plate is welded to a rotationally rigid support e_b is the value obtained from Eq. 6.
If the single plate is welded to a rotationally flexible element, e_b is the value obtained from Eq. 7:

2. Calculate required gross area of plate:

$$A_{vg} \geq R \,/\, 0.40F_y \tag{17}$$

Use A36 steel and select a plate satisfying the following requirements:

a. l_h and $l_v \geq 1.5d_b$. (18)
b. $L_p \geq 2a$ (19)
c. $t_p \leq d_b/2 + 1/16$ (20)
d. $t_p \geq A_{vg}/L_p$ (21)
e. Bolt spacing =3 in.

3. Check effective net section:

Calculate allowable shear strength of the effective net area:

$$R_{ns}=[L_p-n(d_b+1/16)](t_p)(0.3F_u) \quad (22)$$

and satisfy that $R_{ns} \geq R$.

4. Calculate actual allowable shear yield strength of the selected plate:

$$R_o = L_p t_p\ (0.40F_y) \quad (23)$$

Design fillet welds for the combined effects of shear R_o and moment $R_o e_w$ using Table XIX of the AISC Manual.[13] e_w is given in Eq. 16 as:

$$e_w = \text{Max} \left| \begin{array}{ll} (n)(1.0) & (16a) \\ a & (16b) \end{array} \right.$$

The weld is designed for a capacity of R_o, and not for the applied R, to ensure that the plate yields before the welds. However, for A36 steel and E70 electrodes the weld size need not be larger than ¾ of the plate thickness.

5. Check bearing capacity of bolt group:

$$(n)(t)(d_b)(1.2F_u) \geq R \quad (24)$$

If the bolts are expected to resist a moment (as they normally would), this calculation should reflect the reduced strength as determined by Table X of the AISC Manual[13] as demonstrated in the following examples.

6. If the beam is coped, the possibility of block shear failure should be investigated.

Application to Design Problems

The following examples show how the design procedure can be implemented into the design of steel structures.

Design Example 1

Given:

Beam:	W27 × 114, t_w = 0.570 in.
Beam Material:	A36 steel
Support:	Column flange (Assumed rigid)
Reaction:	102 kips (Service Load)
Bolts:	⅞ in. dia. A490-N (snug tight)
Bolt Spacing:	3 in.
Welds:	E70XX fillet welds

Design a single plate framing connection to transfer the beam reaction to supporting column.

Solution:

1. Calculate number of bolts:

Shear = R = 102 kips

Let us assume M = 0, (will be checked later)

$n = R/\,r_v = 102/16.8 = 6.1$

Try 7 bolts

The distance between the bolt line and the weld line a is selected equal to 3 in.

Check moment:

$e_b = (n-1)1.0 - a = 7 - 1 - 3 = 3.0$ in.

Moment = 3 × 102 = 306 kip-in.

Using Table X of the AISC-ASD Manual[13] with eccentricity of 3 in., a value of 6.06 is obtained for effective number of bolts (7 bolts are only as effective as 6.06 bolts).

Therefore,

$R_{bolt} = 6.06 \times 16.8 = 101.8 \approx 102$ kips **O.K.**

Use: Seven ⅞ in. dia. A490-N bolts.

2. Calculate required gross area of the plate:

$A_{vg} = R\,/\,0.40F_y$

$A_{vg} = 102/(0.40 \times 36) = 7.08$ in.^{2}

Use A36 steel and select a plate satisfying the following requirements:

a. l_h and $l_v \geq 1.5d_b$

$l_h = l_v = 1.5(7/8) = 1.32$ in.

$W = a + l_h = 3 + 1.32 = 4.32$; **use W = 4½ in.**

b. $L_p/a \geq 2.0$

$L_p = 2 \times 1.32 + 6 \times 3.0 = 20.6$ in.; **use L_p = 21 in.**

Check: $L_p/a = 21/3 = 7 > 2$ **O.K.**

c. $t_p \leq d_b/2 + 1/16$

$t_p \leq (7/8)/2 + 1/16 = 1/2$ in.

d. $t_p = A_{vg}\,/L_p$

$t_p = 7.08/21 = 0.337$ in.

Try PL 21 × ⅜ × 4-½

3. Calculate allowable shear strength of the net area:

$R_{ns} = [L_p-n(d_b + 1/16)](t_p)(0.3F_u)$

$R_{ns} = [21-7(7/8 + 1/16)](3/8)(0.3 \times 58) = 94 < 102$ kips **N.G.**

Try ½ in. thick plate:

$R_{ns} = [21-7(7/8 + 1/16)](1/2)(0.3 \times 58) = 125 > 102$ kips. **O.K.**

Use: PL 21×½×4½, A36 Steel.

4. Calculate the actual allowable yield strength of the selected plate:

$R_o = L_p t_p\ (0.40F_y)$

$R_o = 21 \times 0.5 \times 0.40 \times 36 = 151$ kips

Design fillet welds for the combined effects of shear and moment:

Shear = R_o = 151 kips

$$e_w = \text{Max} \left| \begin{array}{ll} n(1.0) = 7(1.0) = 7 \text{ in.} \\ a \qquad\qquad\qquad = 3 \text{ in.} \end{array} \right.$$

Therefore, e_w = 7.0 in.

Moment = $R_o e_w$ = 151 × 7 = 1057 kip-in.

Using Table XIX AISC Manual[13]

$a = 7/21 = 0.333$
$C_1 = 1.0$
$C = 1.07$
$D_{16} = R_o/CC_1L_p = 151/(1.0 \times 1.07 \times 21) = 6.72$
Since weld size need not be greater than $0.75t_p$,

Use: ⅜ in. E70 Fillet Welds.

5. Check bearing capacity:
   For plate:
   $r_v = d_bt_p\ (1.2F_u) = .875 \times .5 \times 1.2 \times 58 = 30.45$

   $R_{brg} = 6.06(30.45) = 184.5 \text{ kips} > 102 \text{ kips}.$ **O.K.**
   Since the beam web is thicker than the plate, the web will not fail.

6. Beam is not coped, therefore, there is no need for consideration of block shear failure.

Design Example 2

Given:

Beam:	W16×31, $t_w = 0.275$
Beam Material:	A572 Gr. 50 steel
Support:	Condition of support is unknown
Reaction:	33 kips (Service Load)
Bolts:	¾ in. dia. A325-N or A490 (snug tight)
Bolt Spacing:	3 in.
Welds:	E70XX fillet welds

Design a single plate shear connection to transfer the beam reaction to the support.

Solution:

1. Calculate number of bolts:
   Shear = 33 kips
   Let us assume $M = 0$, (will be checked later)
   Try A325-N bolts with 9.3 kips/bolt shear capacity:
   $n = R/\ r_v = 33/9.3 = 3.5$
   Try 4 bolts.
   The distance between bolt line and weld line a is selected equal to 3 in.
   Check moment:
   Since condition of support is not known, the support is conservatively assumed to be flexible for bolt design. Therefore e_b is equal to 3 in.
   Moment = 3 × 33 = 99.0 kip-in.
   Interpolating from Table X[13], $C \approx 2.81$
   $R_{all} = 2.81 \times 9.3 = 26.1 \text{ kips} < 33$ **N.G.**
   Which indicates 4 A325 bolts are not enough. Let us try 4 A490-N bolts:
   $R_{all} = 2.81 \times 12.4 = 34.8 \text{ kips} > 33$ **O.K.**

   Use: Four ¾ in. dia. A490-N bolts.

2. Calculate required gross area of plate:
   $A_{vg} = R\ /\ 0.40F_y$
   $A_{vg} = 33/(0.40 \times 36) = 2.29 \text{ in.}^2$

Use A36 steel and select a plate satisfying the following requirements:

   a. l_h and $l_v \geq 1.5d_b$.
      $l_h = l_v = 1.5(3/4) = 1.125$ in.
      $W = a + l_h = 3 + 1.125 = 4.125$ in.
      Use: $W = 4\frac{1}{2}$ in.
   b. $L_p/a \geq 2.0$
      $L_p = 3 + 3 \times 3 = 12$ in.
      Check: $L_p/a = 12/3 = 4 > 2$ **O.K.**
   c. $t_p \leq d_b/2 + 1/16$
      $t_p \leq (3/4)/2 + 1/16 = 7/16$ in.
   d. $t_p = A_{vg}/L_p$
      $t_p = 2.29/12 = 0.19$ in.

Use: PL 12×¼×4½, A36 Steel.

3. Calculate allowable shear strength of the net area:
   $R_{ns} = [L_p - n(d_b + 1/16)](t_p)(0.3F_u)$
   $R_{ns} = [12 - 4(3/4 + 1/16)](1/4)(0.3 \times 58) = 38.1$ kips
   $R_{ns} \geq R$ is satisfied.

4. Calculate actual allowable yield strength of the selected plate:
   $R_o = L_pt_p\ (0.40F_y)$
   $R_o = 12 \times 0.25 \times 0.40 \times 36 = 43.2$ kips
   Design fillet welds for the combined effects of shear and moment:
   Shear $= R_o = 43.2$ kips

   $$e_w = \text{Max} \left| \begin{array}{ll} (n)(1.0) = 4(1.0) = 4 \text{ in.} \\ a = 3.0 \end{array} \right.$$

   Therefore, $e_w = 4.0$ in.
   Moment $= R_oe_w = 43.2 \times 4 = 172.8$ kip-in.
   Using Table XIX AISC Manual[13]
   $a = 4/12 = 0.33$
   $C_1 = 1.0$
   $C = 1.07$
   $D_{16} = R_o/CC_1L_p = 43.2/(1.0 \times 1.07 \times 12) = 3.36$
   Since weld size need not be greater than $0.75t_p$,

   Use: $\frac{3}{16}$ in. E70 Fillet Welds.

5. Check bearing capacity.
   For plate:
   $nd_bt_p\ (1.2F_u) = 2.81 \times .75 \times .25 \times 1.2 \times 58$
   $= 36.7 \text{ kips} > 33 \text{ kips}.$
   and for beam:
   $nd_bt_w(1.2F_u) = 2.81 \times .75 \times .27 \times 1.2 \times 65$
   $= 44.4 \text{ kips} > 33 \text{ kips}.$

6. Beam is not coped, therefore, no need for consideration of block shear failure.

CONCLUSIONS

Based on the studies reported here, the following conclusions were reached:

1. The experimental studies of single plate connections in-

dicated that considerable shear and bearing yielding occurred in the plate prior to the failure. The yielding caused reduction of the rotational stiffness which in turn caused release of the end moments to midspan of the beam.

2. The limit states associated with single plate connections are:
   a. Plate yielding.
   b. Fracture of net section of plate.
   c. Bolt fracture.
   d. Weld fracture.
   e. Bearing failure of bolt holes.
3. A new design procedure for single plate shear connections is developed and recommended. The procedure is based on a concept that emphasizes facilitating shear and bearing yielding of the plate to reduce rotational stiffness of the connection.
4. To avoid bearing fracture, the horizontal and vertical edge distance of the bolt holes are recommended to be at least 1.5 times diameter of the bolt. The study reported here indicated that vertical edge distance, particularly below the bottom bolt is the most critical edge distance.
5. Single plate connections that were tested were very ductile and tolerated rotations from 0.026 to 0.061 radians at the point of maximum shear. Rotational flexibility and ductility decreased with increase in number of bolts.

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NOMENCLATURE

A_{ns} Net area in shear, in.^{2}
A_{nse} Effective net area of plate in shear, in.^{2}
A_{vg} Gross area of plate in shear, in^2.
C Coefficient in the AISC Manual Tables X and XIX
C_1 Coefficient in the AISC Manual Table XIX
D_{16} Number of sixteenth of an inch in fillet weld size
F_u Specified minimum tensile strength of steel, ksi
F_{vy} Allowable shear stress for plate in yielding = $0.40F_y$, ksi
F_{vu} Allowable ultimate shear strength = $0.30F_u$, ksi
F_y Specified yield stress of steel, ksi
L Length of span, in.
L_p Length of plate, in.
M_p Plastic moment capacity of cross section = Z_xF_y
M_y Yield moment of beam cross section, kip-in.
R Reaction of the beam due to service load, kips
R_{bolt} Allowable shear capacity of bolt group
R_{ns} Allowable shear fracture capacity of the net section
R_o Allowable shear yield strength of plate, kips
R_y Reaction corresponding to plastic collapse of beam, kips
S_x Section modulus in.^{3}
V Shear force, kips
W Width of plate, in.
Z_x Plastic section modulus, in.^{3}
a Coefficient in the AISC Manual Table XIX
a Distance between bolt line and weld line, in.
d Depth of beam, in.
d_b Diameter of bolt, in.
e Eccentricity of point of inflection from the support
e_b Eccentricity of beam reaction from bolt line, in.
e_w Eccentricity of beam reaction from weld line, in.
f_{vy} Computed shear stress in plate gross area, ksi
f_{vu} Computed shear stress in plate effective net area, ksi
l_h Horizontal edge distance of bolts, in.
l_v Vertical edge distance of bolts, in.
n Number of bolts
r_v Allowable shear strength of one bolt, kips
t_p Thickness of plate, in.
t_w Thickness of beam web, in.